BLAST INDUCED SOIL LIQUEFACTION
- State-of-the-Art -

Wayne A. Charlie, Associate Professor
George E. Veyera, Instructor
Steven R. Abt, Associate Professor
Howard D. Patrone, Student Assistant

Civil Engineering Department
Colorado State University
Fort Collins, Colorado, U.S.A.

ABSTRACT

This paper reviews blast induced soil liquefaction and describes an experimental laboratory testing program being conducted in the Civil Engineering Department's Geotechnical Engineering Laboratory at Colorado State University. The study of the behavior of water saturated sands under shock loadings is being conducted to evaluate potential blast induced changes in dynamic soil properties and soil shear strength loss (liquefaction). The facility is capable of generating single and multiple shock pulses with milli-second rise times, peak stress amplitudes of up to 35,000 KPa (5000 psi), peak particle velocities of 1000 cm per second (400 in. per sec.) and peak accelerations of 2,000 g. Of major interest is the behavior of the water pressure in the soil, both during and after the passage of the stress wave, as a function of strain, soil density, initial confining stress and the number of loadings. The information gained from the experiments will assist in improving ground shock prediction techniques for water saturated sands.

INTRODUCTION

Engineering designs presently incorporate the assumption of little or no blast induced soil property changes. However, evidence indicates that blast induced soil property changes, such as changes in shear strength, shear wave velocity, damping and water pressure are likely to have occurred at some test sites having loose saturated granular soils (Charlie et al., 1981). For porewater pressure response, the three stages of interest which may occur as a result of blasting are the milli-second transient response directly associated with the passage of the stress wave, the residual response shortly after the passage of the stress wave, and the longer term dissipation of the residual porewater pressures. Blast induced residual porewater pressure increases have been reported by Florin and Ivanov (1961), Kummeneje and Eide (1961), Terzaghi (1965), Damitio (1972), Langley et al. (1972), Perry (1972), Banister and Ellett (1974), Yamamura and Koga (1974), Charlie (1977), Rischbieter (1977), Arya et al. (1978), Charlie (1978), Damitio (1978), Kok (1978), Marti (1978), Studer and Kok (1980), Long et al. (1981), Prakash (1981), and other researchers. Such soil behavior may indicate that liquefaction, described as a process in which a saturated cohesionless soil loses shear strength as a result of increased pore pressures, may have occurred at these sites. As such, an explosion detonated in a soil having a high liquefaction potential could result in damage disproportionate to the energy released.

A study of the behavior of saturated cohesionless soils under blast loadings was initiated at Colorado State University in 1978. This study has resulted in the development of a laboratory testing facility capable of subjecting a saturated soil sample to single and multiple shock pulses with milli-second rise times.

ASSESSMENT OF PREDICTION METHODS

The state-of-the-art for assessing blast induced residual porewater pressure increases and liquefaction potential is limited at best. Theoretical approaches are almost non-existent and have not been verified by experimental testing. Empirical scaling factors have been derived from a limited number of field tests. A logical approach would be to determine possible threshold particle velocities, stresses or strains below which blast induced porewater pressure increases should not occur. Lyakhov (1961) noted that blast induced liquefaction did not occur in water saturated sand with densities greater than 1.6 gm per cubic cm. For saturated soils at lower densities, Puchkov (1962) found that soils did not liquefy below a peak particle velocity of 4 cm per second. Damitio (1978) and Kok and Studer (1980) have reported empirical relationships to predict the maximum radius of liquefaction from contained point charges. These relationships for loose saturated sands indicate liquefaction may occur above particle velocities of 4 cm per second. Obermeyer (1980) measured no significant increase in residual porewater pressures in a hydraulic fill tailings dam subject to blast generated peak particle velocities of 2 cm per second. Several earthfill dams have been subjected to subsurface nuclear detonations including Navajo Dam, New Mexico (peak particle velocity of 1.3 cm per second) and Rifle Gap Dam (peak particle velocity of 2.5

cm per second). Measurements taken a few hours after the tests showed little or no increase in porewater pressures (Rouse et al., 1970; Ahlberg et al., 1972).

Increased residual porewater pressure increases were measured in a riverbed consisting of clayey silty soils subjected to peak particle velocities exceeding 11 cm per second from an underground nuclear explosion (Banister and Ellett, 1974). Residual porewater pressures from other field explosive tests have been reported at peak particle velocities as low as 1 cm per second. Marcuson (1982) suggests that liquefaction should not occur where the peak particle velocity is less than 2.5 cm per second. Sanders (1982) and Seed (1982) related earthquake induced liquefaction to peak particle velocity indicating that a threshold particle velocity of 5 to 10 cm per second was a value that may also hold for blasts. For a 500 ton TNT surface explosion, Langley et al. (1972) measured up to 45 KPa residual porewater pressure increases out to distances of 170 meters from the detonation point. The estimated peak airblast over pressure at 170 meters was 2000 KPa. Perry (1972) conducted shock tube tests and determined that a loose saturated sand could be liquified at peak over pressures as low as 250 KPa.

PROPOSED THRESHOLD STRAIN APPROACH

A threshold strain approach may prove very useful for assessing blast induced residual porewater pressure increases since shear or compression strain of less than 10^{-2} percent is generally considered not to generate residual porewater pressures upon unloading since strains are in the elastic range (Dobry et al., 1982). Utilizing equations given by Rinehart (1975) and Richart et al. (1970), Table 1 shows that a compression strain of 10^{-2} percent in water saturated soils at a void ratio equal to one corresponds to about 15 cm per second peak longitudinal particle velocity. Shear strains of 10^{-2} percent correspond to about 1 cm per second peak transverse particle velocity for soils at a void ratio equal to one located near the ground surface. Based on scaling factors given by Dupont (1980), for a single contained detonation of 100 kg of explosives, peak particle velocities would exceed 1 cm per second within 200 meters from the detonation point. Table 2 presents several empirical scaling factors based on threshold strain, particle velocity and field tests to determine the potential radius of liquefaction and residual porewater pressure increases for various charge weights. Although there are differences in the predictions, the predicted residual porewater pressure increases occur at distances greater than is generally considered to be hazardous to blast resistant structures. Based on the Russian research with multiple charges, unless complete dissipation of the residual porewater pressure can occur between stress waves or between detonations, the predicted maximum radius of residual porewater pressure increases may be greater than that given in Table 2.

LABORATORY FACILITY AND INSTRUMENTATION

Laboratory testing is currently being conducted at Colorado State University to evaluate the empirical scaling factors listed in Table 2, to evaluate the effects of multiple shock loadings, and to extend the state-of-the-art in understanding stress wave mechanics of two phase materials. The objectives of the testing program are to generate and determine the number of axial compressive stress pulses required to induce liquefaction in saturated cohesionless soils as a function of:

- Initial Relative Density
- Initial Effective Stress
- Peak Particle Velocity
- Peak Strain Amplitude
- Peak Stress Amplitude

Data collection objectives include measuring and recording both transient and long term response of the soil's:

- Porewater Pressure
- Particle Velocity
- Strain
- Stress

The soil is being tested in an undrained state since little or no short term drainage would occur in deep field deposits of saturated soils.

The laboratory shock facility, shown schematically in Figure 1, consists of separate but intimately related elements which include a gas-charged cannon, two fluid filled stainless steel tubes between which the soil sample is placed, a rigid stainless steel sample container, flexible membranes, an electronic control system, and an electronic monitoring and recording system. The membranes are utilized to apply the confining pressure and allow the soil to be tested in an undrained state. The cannon is designed to fire 7 cm diameter projectiles of various masses which impact a piston at the end of the fluid filled stainless steel impact tube. The piston imparts a stress wave to the fluid which then transmits a compressive shock pulse to the soil sample.

A pressure transducer, positioned just upstream of the sample is used to determine the intensity of the stress imparted to the sample. A second pressure transducer measures the pressure in the fluid just downstream of the sample, and a third pressure transducer measures porewater pressures in the soil. The transducers measure both the peak and long-term porewater pressure response. The pressure transducers are ENDEVCO Model 8511A-5K which have a porous metal. The resonant frequency is greater than 500 k Hz over a dynamic range of 0 to 35,000 KPa. Although particle velocities and strains in the sample have not been measured to date, inductance strain gages with very small masses (Bison Model 4104) and accelerometers are currently being evaluated. To minimize reflections, an energy trap, which consists of a 10 cm diameter, three meter long solid

polyvinyl chloride (PVC) bar, is utilized. In current testing the energy trap is placed at the end of the sample.

Other instrumentation consists of a set of three signal conditioners (ENDEVCO Model 4470), three amplifiers (ENDEVCO Model 4476.1A), a dynamic strain gage (Bison Model 4101A), a four channel high speed digitizer (Biomation Model 2805), a desk top computer (Hewlett Packard Model 9835) and a plotter (Hewlett Packard Model 9872C). A time interval counter (Hewlett Packard Model 5300) is used to determine the impact velocity of the projectile.

LABORATORY RESULTS

Tests are being conducted on saturated sand at densities ranging from 1.4 to 1.7 gm per cubic cm (0 to 100 percent relative density) with initial effective stresses from 100 to 1000 KPa. This stress range corresponds to the effective vertical geostatic stress at a depth of approximately 5 to 100 meters below the ground surface. All tests to date have been conducted in water saturated Montery No. 0/30 (Muzzy, 1983) sand at a dry density of 1.47 gm per cubic cm under initial effective confining stresses of 170, 345 and 690 KPa.

Figures 2, 3 and 4 give the preliminary results of shock testing of two samples of saturated Monterey No. 0/30 sand placed at a dry density of 1.47 gm per cubic cm. In each of these figures, part (a) shows the impact stress in the fluid upstream of the sample, part (b) shows the sample's porewater pressure response to this impact stress, and part (c) combines both the input shock and the sample's porewater pressure response. For the sand under an initial effective stress of 170 KPa, Figure 2 shows that the sample liquefied under one shock loading of 4000 KPa. For an identical sample under an initial effective stress of 690 KPa, Figure 3 shows that the sample's residual porewater pressure increased by 200 KPa after being subjected to a peak input stress of 2000 KPa. This porewater increases is about 30 percent of the increase required to cause liquefaction. Figure 4 shows that the same sample liquefied when subjected to a second shock of 4100 KPa. The preliminary results of these and other tests are summarized in Table 3. The relationships given earlier in this paper predict residual porewater pressure increases at these impact stresses and strains.

CONCLUSIONS

Today's understanding of blast induced liquefaction has advanced only slightly beyond the point of recognition of its existence. Documented occurence, although sketchy and often incomplete, is available in the open literature. Although considerable work remains to be done in projecting this information into a comprehensive method of predicting liquefaction for actual or hypothetical blasts, the limited data indicates that residual porewater pressure increases should not occur in soils subjected to strains less than 10^{-2} percent. The laboratory facility described in this paper is assisting in developing testing techniques for the evaluation of blast induced liquefaction potential. The data will also be useful in verifying and developing empirical correlations and mathematical models.

ACKNOWLEDGMENTS

Support for this work was in part provided by the Air Force Office of Scientific Research and the Civil Engineering Department at Colorado State University. The writers acknowledge and appreciate the technical assistance received from Nancy C. Patti of Science Applications Inc., Steamboat Springs, Colorado, S. Melzer of Civil Systems, Inc., Midland, Texas, and J. Shinn of Applied Research, Inc., S. Royalton, Vermont, and M. W. Muzzy of Colorado State University. The writers are also grateful to C. Emanuel who typed the manuscript.

REFERENCES

1. Ahlberg, J. E., Fowler, J. and Heller, L. W. (1972), "Earthquake Resistance of Earth and Rock Fill Dams: Analysis of Response of Rifle Gap Dam to Project Rulison Underground Nuclear Detonation," U.S. Army WES, Misc. Paper 5-71-17, June.

2. Arya, A. S., Nandakumaran, P., Puri, V. K. and Mukerjee, S. (1978), "Verification of Liquefaction Potential by Field Blast Tests," Proc. of the 2nd Int. Conf. on Microzonation, Vol. II, San Francisco, California, November-December.

3. Banister, J. R. and Ellett, D. M. (1974), "Pore Pressure Enhancement Observation on Rio Blanco," Report SLA-74-032, Test Effects Department 1150, Sandia Laboratories, Albuquerque, New Mexico, August.

4. Charlie, W. A. (1977), "Liquefaction Potential of Soils Under Blast Loads," Air Force Office of Scientific Research, Report No. 46, Volume 2, AFOSR-TR-78-0349.

5. Charlie, W. A. (1978), "The Dial Pack Event," Int. Workshop on Blast-Induced Liquefaction, Organized by Dames and Moore, London, and sponsored by U.S. Air Force, Maidenhead, U.K., September.

6. Charlie, W. A., Mansouri, T. A. and Ries, E. R. (1981), "Predicting Liquefaction Induced by Buried Charges," Proc. of the Int. Conf. on Soil Mechanics and Foundation Eng., Stockholm, June.

7. Crawford, R. E., Higgins, C. J. and Bultmann, E. A. (1974), The Air Force Manual for Design and Analysis of Hardened Structures, AFWL-TR-74-102, Air Force Systems Command, Kirtland AFB, NM, October.

8. Damitio, C. (1972), "La Consolidation des Sols Sans Cohesion par Explosion," Construction, Vol. 27, No. 3 (In French), France.

9. Damitio, C. (1978), "Field Experience on Blast-Induced Liquefaction," Int. Workshop on Blast-Induced Liquefaction, Organized by Dames and Moore, London, and sponsored by U.S. Air Force, Maidenhead, U.K., September.

10. Dobry, R., Ladd, R. S., Yokel, F. Y., Chung, R. M. and Powell, D. (1982), "Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by Cyclie Strain Method," NBS Building Science Series 138, National Bureau of Standards, July.

11. Dupont (1980), Blasters Handbook, 16th Edition, E.I. duPont de Nemours and Co., Explosive Products Division, Wilmington, Delaware.

12. Florin, V. A. and Ivanov, P. L. (1961), "Liquefaction of Saturated Sandy Soils," Proc. 5, Int. Conf. Soil Mechanics and Foundation Engineering, ICSMFE, Vol. 1.

13. Kok, L. (1978), "Empirical Predictions of Blast-Induced Liquefaction," Proc. Int. Workshop on Blast-Induced Liquefaction, Dames and Moore/U.S. Air Force, Maidenhead, U.K.

14. Kummeneje, D. and Eide, O. (1961), "Investigation of Loose Sand Deposites by Blasting," Proc. 5th ICSMFE, Vol. 1.

15. Langley, N. P., et. al. (1972), "Dial Pack Event - Soil Pore Pressure and Shear Strength Test," Aerospace Corp. Report No. TOR-0172 (S2970-20)-1, February.

16. Long, J. H., Ries, E. R. and Michalopoulos, A. P. (1981), "Potential for Liquefaction Due to Construction Blasting," Proc. Int. Conf. on Recent Advances in Geotech. Eng. and Soil Dynamics, Univ. of Missouri-Rolla, St. Louis, Missouri.

17. Lyakhov, G. M. (1961), "Shock Waves in the Ground and the Dilatancy of Water Saturated Sand," Zhurnal Prikladnoy Mekhanik; i Tekhnicheskoy Fiziki, Vol. 1., pp. 38-46, Moscow.

18. Marcuson, W. (1982), U.S. Army Waterways Experiment Station, Vicksburg, Miss., Personal Communication, July.

19. Marti, J. (1978), "Blast-Induced Liquefaction: A Survey," Report by Dames and Moore for the U.S. Air Force Office of Scientific Research, Report No. TR-78-3, May.

20. Muzzy, M. W. (1983), "Cyclic Triaxial Behavior of Monterey No. 0 and No. 0/30 Sands," M.S. Thesis, Colorado State University, Department of Civil Engineering.

21. Obermeyer, J. R. (1980), "Monitoring Uranium Tailing Dams During Blasting Program," Symp. on Uranium Mill Tailings Management, Colorado State University, Fort Collins, CO, Nov.

22. Perry, E. B. (1972), "Movement of Variable-Density Inclusions in Wet Sand Under Blast Loading," U.S. Army Eng. Waterways Experiment Station, Misc. Paper S-72-37, September.

23. Prakash, S. (1981), Soil Dynamics, McGraw-Hill Book Company, N.Y.

24. Puchkov, S. V. (1962), "Correlation Between the Velocity of Seismic Oscillations of Particles and the Liquefaction Phenomenon of Water-Saturated Sand," Issue No. 6, Problems of Engineering Seismology, Study No. 21, Edited by S. v. Medvedev, translated by Consultants Bureau, New York.

25. Rischbieter, F. (1977), "Soil Liquefaction: A Survey of Research," Proc. of the Fifth Int. Symp. on Military Applications of Blast Simulation, Organized by the Royal Swedish Fortifications Administration, Stockholm, Sweden.

26. Richart, F. E., Hall, J. R. and Woods, R. D. (1970), Vibrations of Soils and Foundations, Prentice-Hall, N.J.

27. Rinehart, J. S. (1975), Stress Transients in Solids, Hyperdynamics, Santa Fe, New Mexico.

28. Rouse, G. C., Roehm, L. H. and Cozart, C. W. (1970), "Vibration of Vega Dam Following Rulison Subsurface Nuclear Blast," U.S. Dept. of Interior, Bureau of Reclamation, REC-OCE-70-15, April.

29. Sanders, S. G. (1982), "Assessment of the Liquefaction Hazards Resulting from Explosive Removal of the Bird's Point - New Madrid Fuze Plug Levee," Misc. Paper GL-82-5, U.S. Army Engineer WES, Vicksburg, Miss., April.

30. Seed, H. B. (1982), Letter and review to S. G. Sandeers, WES, dated March 16, 1982. Letter and review printed in WES Misc. Paper GL-82-5 referenced to Sanders (1982) above.

31. Studer, J. and Kok, L. (1980), "Blast-Induced Excess Porewater Pressure and Liquefaction Experience and Application," Int. Symp. on Soils under Cyclic and Transient Loading, Swansea, U.K., Jan.

32. Terzaghi, K. (1956), "Varieties of Submarine Slope Failures," Proc. of the Eighth Texas Conf. on Soil. Mech. and Fdn. Eng., Univ. of Texas, Austin, Sept. and Harvard Soil Mechanics Series, No. 52, Cambridge, Mass.

33. Yamamura, K. and Koga, Y. (1974), "Estimation of Liquefaction Potential by Means of Explosion Test," Proc. of the Sixth Joint Panel Conf. of the U.S.-Japan Coop. Program in Natural Resources, National Bureau of Standards, May.

Table 1. Predicted Peak Compression Strains and Peak Compressive Stresses vs. Peak Longitudinal Particle Velocities for Loose Saturated Sands.

Peak Longitudinal Particle Velocity (cm/sec)	Peak Longitudinal Particle Velocity (ips)	Peak Compression[1] Strain (Percent)	Peak Compression[1] Stress (psi)	Peak Compression[1] Stress (KPa)
1.5	0.6	1×10^{-3}	6	40
2.5	1	2×10^{-3}	10	70
5	2	3×10^{-3}	20	140
10		7×10^{-3}	40	280
15	6	1×10^{-2}	60	400
25	10	2×10^{-2}	100	700
150	60	1×10^{-1}	600	4000

(1) Assumptions: Loose saturated sand and void ratio = 1.

Rinehart (1975) for relations between particle velocity, strain and stress. Richart et al. (1970) for density and compression wave velocity.

Table 3. Summary of Residual Porewater Pressure Increases in Monterey No. 0/30 Sands Subjected to Shock Loadings.

Sample	Density (gm/cm³)	Initial Effective Confining Stress (KPa)	Impact Number	Peak Impact Stress (KPa)	Estimated[1] Peak Strain (Percent)	Residual[2] Porewater Pressure Increase Ratio (Percent)
4/23/83	1.47	170	1	4000	0.1	100
4/25/83	1.47	345	1	1400	0.04	60
4/25/83	1.47	345	2	1400	0.04	100
4/27/83	1.47	690	1	2000	0.05	30
4/‾ 83	1.47	690	2	4100	0.1	100
(3)	1.47	200	1	> 14000	> 0.4	100

(1) Estir ted from Table 1.

(2) Porewater pressure increase ratio = $\frac{\text{Residual Porewater Pressure Increase}}{\text{Initial Effective Confining Stress}}$

Zero for zero increase in residual porewater pressure.
One hundred percent for liquefaction.

(3) Seve-al tests conducted in late 1982.

Table 2. Predicted Maximum Radius of Liquefaction and Residual Porewater Pressures for Detonation of Contained Point Charges in Loose Saturated Cohesionless Soils.

Charge Weight TNT (kg)[4]	Liquefaction[2]: Russian[5] Peak Particle Velocity > 4 cm/sec	Liquefaction[2]: Russian[6] $R_{max} = k_3 C^{1/3}$ Relative 0%	Liquefaction[2]: Russian[6] $R_{max} = k_3 C^{1/3}$ Density 40%	Porewater Pressure Increase[3]: Comp.[7] Strain $> 10^{-2}$%	Porewater Pressure Increase[3]: Shear[8] Strain $> 10^{-2}$%	Porewater Pressure Increase[3]: Liq.[9] Coeffi. > 0.1	Porewater Pressure Increase[3]: Peak[10] Particle Velocity > 1 cm/sec
1	7	25	8	4	24	15	20
10	23	55	20	12	75	20	60
100	75	120	40	40	240	50	200
1,000	230	250	80	120	750	—	600
10,000	750	540	170	400	2400	—	2000
100,000	2300	1200	370	1200	7500	—	6000

Predicted Maximum Radius (meters)[1]

Notes: (1) Predicted maximum radius may be higher under multiple detonations and some geologic and confinement conditions.
(2) Maximum radius for the residual porewater pressure increase equal to the initial effective vertical stress.
(3) Maximum radius for some increase in residual porewater pressure.
(4) For larger charge weights see Sanders (1982) or Crawford et al. (1974) for particle velocity scaling factors.
(5) Dupont (1980) for peak particle velocity scaling factor and Puckhov (1962) for threshold velocity.
(6) Damitio (1972 and 1978) and Marti (1978)
(7) Dupont (1980) and Rinehart (1975) for compression wave velocity of 1500 meters per second and void ratio equal to one.
(8) Dupont (1980) and Rinehart (1975) for shear wave velocity of 150 meters per second and void ratio equal to one.
(9) Kok and Studer (1980)
(10) Dupont (1980)

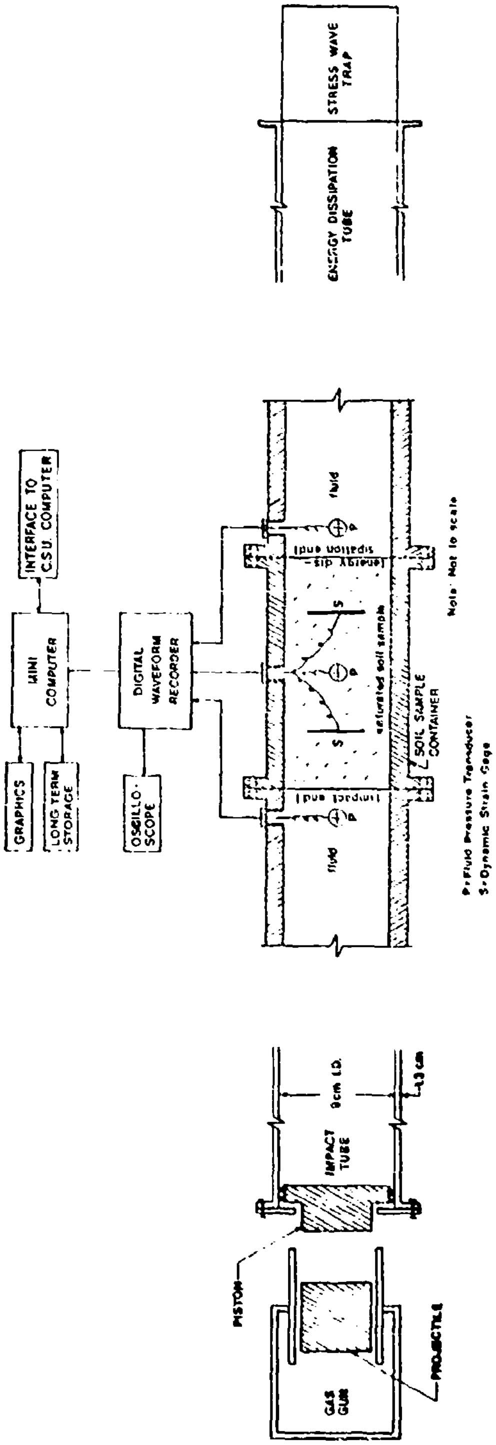

Figure 1. Schematic of shock facility to study blast induced liquefaction.

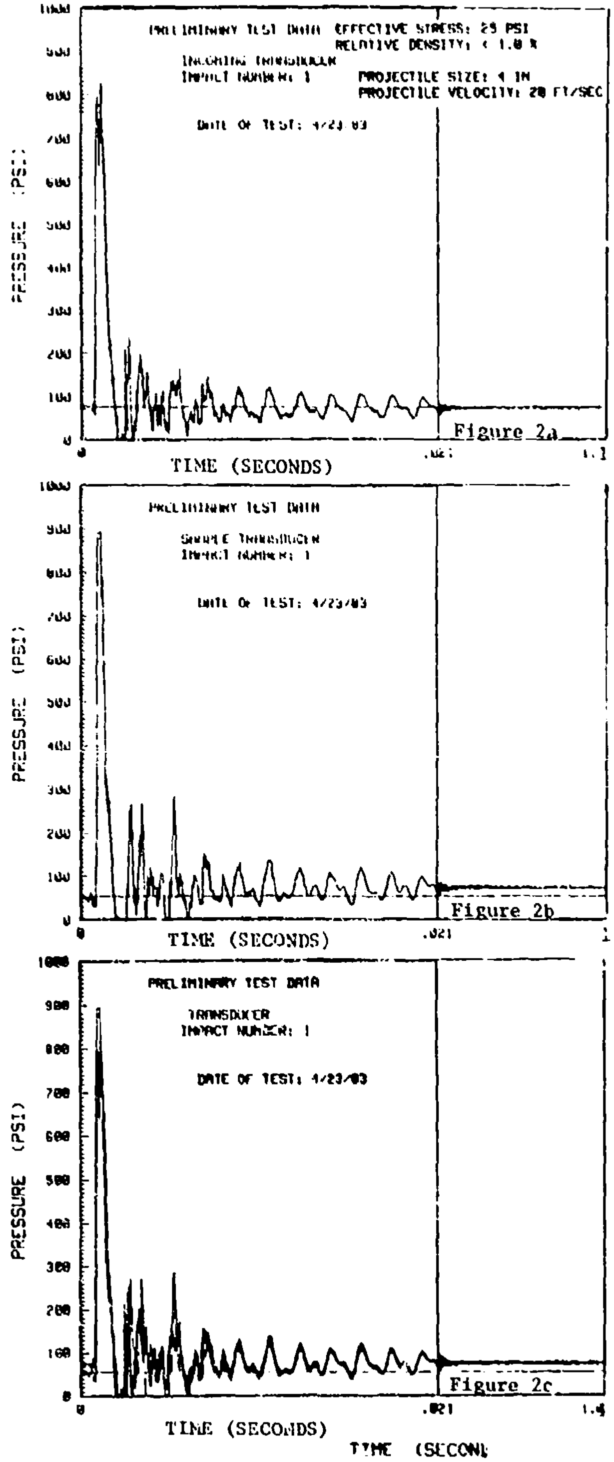

Figure 2. Response of the sample's porewater pressure which lead to liquefaction under one shock loading: initial effective stress of 170 KPa (1 KPa = 6.895 psi).

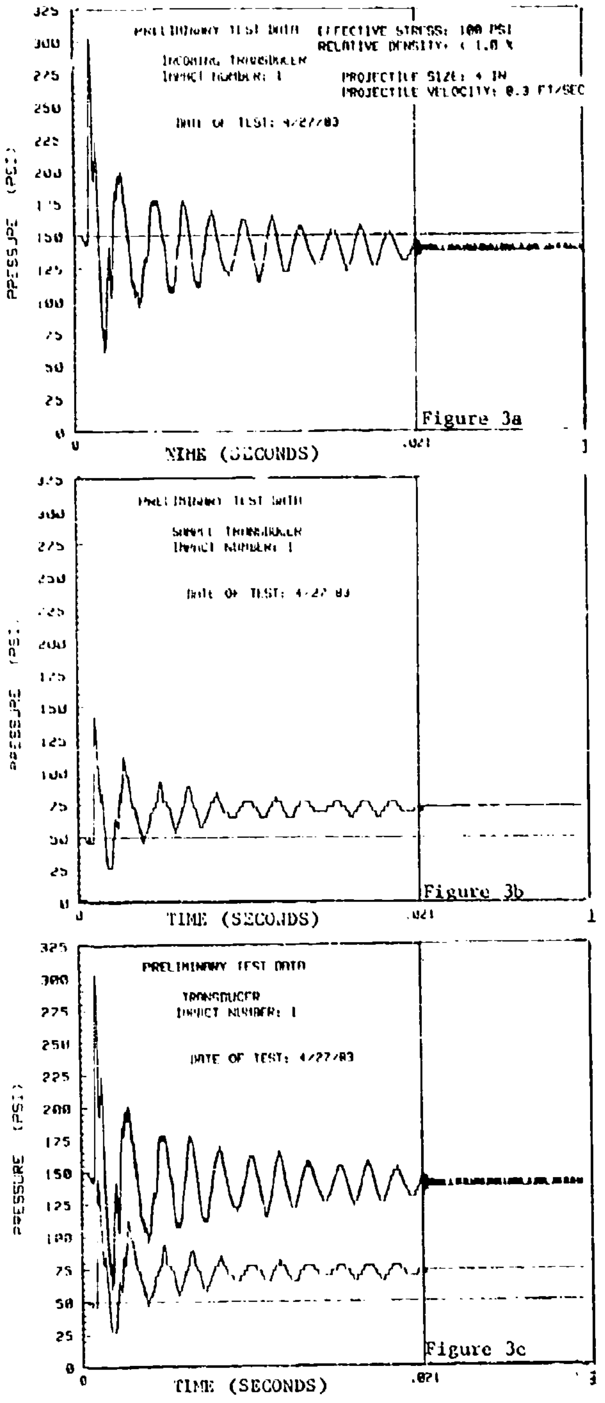

Figure 3. Response of the sample's porewater pressure leading to limited increase in residual porewater pressure under the first shock loading: initial effective stress of 690 KPa (1 KPa = 6.895 psi).

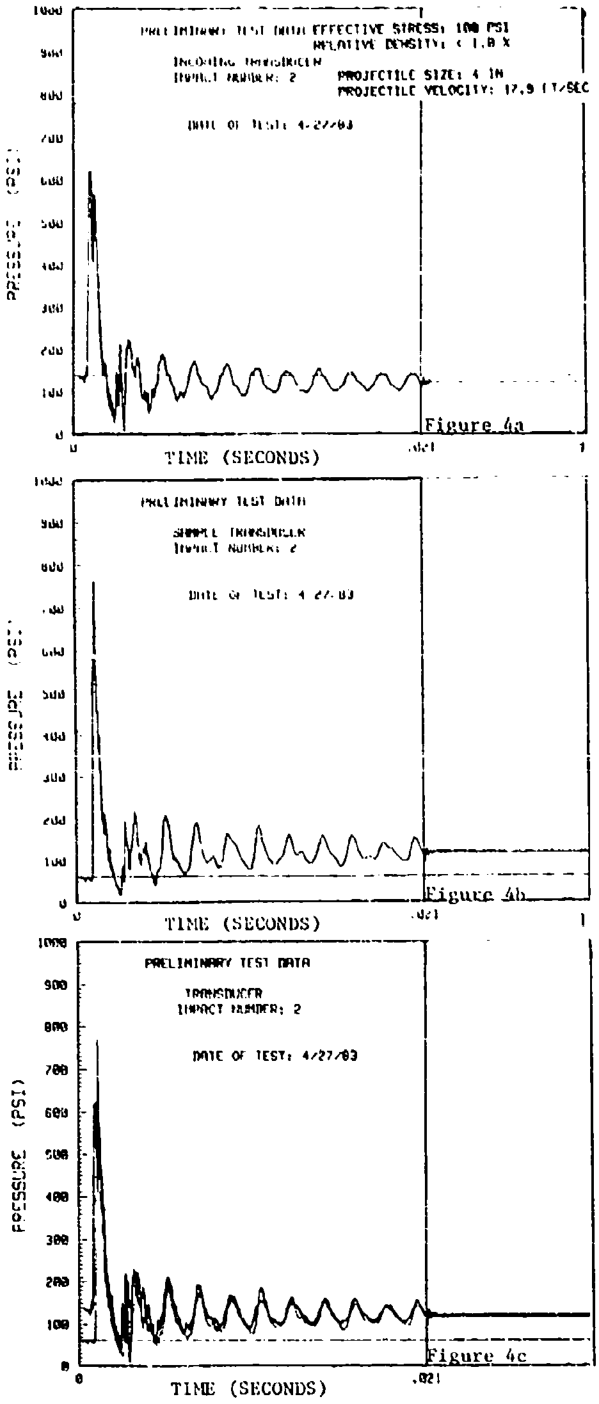

Figure 4. Response of the sample's porewater pressure leading to liquefaction under the second shock loading: initial effective stress of 690 KPa (1 Kp = 5.895 psi).